Validation Isotach model by means of "Schiphol 5th runway"

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Abstract

The report "Validation Isotach model by means of Schiphol 5th runway" describes the comparison between values calculated by the Isotach model with the measurement of settlement and pore pressures during the recent construction of the 5th runway at Schiphol airport. The Isotach model predicts the settlement of a structure, which is founded on soft soil; the settlement is caused by consolidation and creep of the sub soil. The model needs a set of three parameters that can be determined in a Kp-CRS test in the laboratory; the simplicity of this test enables the implementation in every day practice. In order to achieve a considerable reduction in the construction time of the runway, a method of forced drainage was used: the IFCO method. The Isotach model was not yet validated for cases where the soil is subjected to the IFCO method. Such a validation increases the applicability of this model in civil engineering. Therefore the predictions made with the Isotach model were compared with the corresponding measurements of the 5th runway.

Recent work in this project for the Waardse Alliantie has enhanced and improved the Isotach model. The model used an artificial sill stress, which made interpretation ambiguous. As a result the sill stress has been abandoned. Further improvements were reached in the determination of the preconsolidation stress. Analysis of the data as obtained for the "Waardse Alliantie" and for the "Barendrechse weg" reveals the success of this improvement.

The agreement between measured settlement and calculated values is good. Various uncertainties in load and strength contribute to variations in the differences between measurement and calculation. The head in the Pleistocene sand is the major contributor. The second most important contributor is the air pressure in the sand under the foil.

No decisive reasons have been found not to use the Isotach model in situations where intensified consolidation (like IFCO) is used. However a consultant should be aware of the possible deviations between predictions and calculations. Since methods like IFCO use an elaborate monitoring system, possible deviations can be detected in an early stage.

Detailed information on the boundary conditions, like the head in the Pleistocene sand, is necessary to extrapolate or calculate the (remaining) long-term settlement.

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Date: March 2003
Executive Summary

Significance. Roads and railroads are predominantly constructed on an embankment made of locally available sand and clay. The soil beneath the embankment, particularly in the Western and Northern part of the Netherlands, consists of soft and sometimes very soft soils. Large settlements are to be expected. Additionally the low permeability of the soil delays the settlement of a road, since it slows down consolidation. Next to consolidation soft soils are susceptible to creep. Settlement of the road leads to damage, particularly when soft sections of the road are connected to stiffer sections, such as viaducts and bridges. Since the construction process of an embankment takes time, consolidation mainly takes place during the construction of the road, before it fulfils its purpose. Later, when the road is opened for traffic, creep dominates the deformations. These deformations cause mainly financial damage. The organisation that has to maintain the quality of the road has to invest in repair actions. Additionally society looses money due to prolonged periods of additional traffic jams, when the road or part of it, is being repaired.

Modelling. The usual techniques to predict the settlement of a road deal with consolidation and creep in a separate way. The Isotach model deals with them in an integrated way. Changes in the surcharge of the sub soil cause deformation of the soil skeleton. Since soft soils usually have a low permeability, initially the increment in load is translated into an increment of pore pressures, which will gradually dissipate. Simple hydraulic equations using the permeability of the layers of soil deal with the resulting ground water flow. Parameters describing the behaviour of the soil skeleton under compression determine its deformation. Hydraulics and the soil mechanics interact: deformation of the soil skeleton changes the permeability and thus the ground water flow. Assuming that the pore water is hardly compressible, equations for the conservation of mass of the water and for the compressibility of the soil skeleton determine the resulting settlement.

In the previous paragraph it is explained which types of parameters determine the settlement: the mechanical behaviour of the soil skeleton and the hydraulic permeability of the soil. These parameters can both be measured in a Kp-CRS test, in which a sample of soft soil is loaded with a constant rate of strain. The parameter responsible for creep is determined by introducing either a stage with constant stress or with constant strain.

Purpose. The Isotach model can predict settlement of a road, based on laboratory tests, before the road or its embankment, is a tangible object. So the model transcends extrapolation of measured settlement data, as obtained during the construction of a road. The question is: "What is this prediction worth?" In order to ascertain the correctness of the prediction, the Isotach model has to be compared with experimental results: validation. Differences between prediction and measurement reveal the correctness of the model. In stead of predictions one can use postdictions for the purpose of validation as well, as long as one can withstand the temptation to improve the model or its parameters till the point that there is an excellent agreement between measurement and calculation.

Background. Earlier attempts to validate the model have revealed differences between prediction and measurements. The main cause was shown to be the sill stress. This stress has been introduced to quench numerical oscillations at low stresses. The sill stress is an extra parameter, which can not be measured in the laboratory. Additionally the model was shown to be quite sensitive for the sill stress. A physically sound method for the deformation at low stresses has been described by den Haan et al. [2000], but the translation of this physical formulation into a numerical model lacks interests of the market parties. Alternatively the concept of the pre overburden pressure (POP) is introduced. It has shown to be successful for the calculations for the "Waarde Alliantie" and the "Barendrechtse weg" subprojects.

The concept of the strain dependent permeability is another improvement in the implementation of the Isotach model that has been introduced during the work for the "Waarde Alliantie". Due to the deformation of the soil skeleton, the pores become narrower. So when strain increases, permeability
decreases. A further improvement was obtained in the determination of the preconsolidation stress, using natural strain in stead of the conventional method in which linear strain is used.

Scope. All in situ measurements were performed by the contractor that has built the runway. He measured settlement and pore pressures in the soil and in the drainage system. Regarding validation of the Isotach model application of the IFCO method introduces an uncertainty in the total settlement, since measurements before any human activity are not recorded. The IFCO method uses differential methods to ascertain the final settlement. The contractor has reported all data in written reports. Data in digital form is not available. Browsing the available data, two sections were chosen: section A1 and A6. Criterion for this choice was the availability of more or less reliable data.

The geotechnical profile along the runway is well documented. The contractor had performed roughly 150 CPTs and four borings along the projected runway. On samples of the borings he has performed oedometer tests. Since data of the Kcr-CRS tests are not available, GeoDelft has performed an additional boring near section A1. On six representative samples of this boring Kcr-CRS tests have been performed. From these tests the appropriate parameters for the Isotach model have been derived. By comparison of the geotechnical profile with the description of the soil, as obtained by the GeoDelft boring, corresponding layers in the geotechnical profile were identified. The parameters for the Isotach model have been attributed to the observed layers in the contractors geotechnical profile.

Calculation of the settlement was performed by the MSettle suite, version 6.7. The strain dependent permeability is calculated in a spreadsheet. The parameters of the Isotach model have been derived manually and by means of MCompress version 1.0. The value of the preconsolidation stress is determined manually, using natural strain.

Results. Two sections of the runway, i.e. A1 and A6, were chosen for the comparison between measured and postdicted settlement and pore pressure data. For section A1 the agreement between prediction and measurement is very good. For section A6 there is a difference of about 8 cm on a total settlement of 23 cm at the end of the period during which the pressure in the sand under the foil was reduced. The time schedule for the initial construction phases of section A6 however is not know in detail, which may contribute to the observed differences.

The agreement between measured pore pressures and calculated values is poor. Since initial values of pore pressures are not available, the origin of the observed deviations can not be explained in a reliable way. The fact that the average value of the pore pressure is calculated will contribute to differences between calculation and measurement.

Conclusions.
The value of the absolute settlement was not measured. This was not an omission of the contractor, since consultancy related to the IFCO method, uses changes in settlement and pore pressures. However the absence of absolute data implies some limitations on the comparison between measurement and calculations, as described in this report.

The time schedule for section A6 is not known in detail. Although using a similar time schedule as in section A1 is the best possible estimation, this assumption will introduce some uncertainty in the results.

The agreement between measurement and calculation is better for settlement than for pore pressures. A similar result has been found in [den Adel 2003].

Various uncertainties in load and strength contribute to variations in the differences between measurement and calculation. Their influence has been reported in the different cases. From these case is concluded that the head in the Pleistocene sand is the major contributor. The second most important contributor is the air pressure in the sand under the foil.
In which extent the concept of the Isotach model contributes to the observed differences between measurement and calculation is hard to determine. The global idea is that uncertainties in load and strength do have a major influence on the predictions of the model. If these uncertainties can be narrowed to a minor margin, the deviations between predictions and observations will definitely decrease.

This report has shown that for the sections A1 and A6 of the 5th runway, where a method of intensified consolidation (like IFCO) is used, the Isotach model is a good predictor for the settlement. It should be noted that two sources of uncertainty have complicated the comparison between measured and calculated results: the fluctuations in the observed settlement and the absence of measured data in the initial situation.

**Recommendations.**

If the Isotach model is to be used in situations where an intensified or forced system of drainage is a vital part of the construction process, a monitoring system for pore pressures and settlement is of importance.

Measurement of and reporting the initial situation will simplify the understanding of monitoring data.

Detailed information on the boundary conditions, like the head in the Pleistocene sand, is necessary to extrapolate or calculate the (remaining) long term settlement.

Information on the magnitude of the pressure drop over the sealing foil will enhance the accuracy of the (remaining) long term settlement.

Detailed information on the volumetric weight of the soil in the embankment will increase the reliability of the predictions.

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Applicability for the sector

General. The Isotach model for the prediction of settlement has been implemented in the MSettle suite. Analysis of the "Waardse Alliantie" data has shown that the sill stress is not a particularly good parameter, since it influences the calculated settlement in an unacceptable way. Furthermore it was shown that the influence of strain on the permeability of the soil is another parameter which to be accounted for.

Conclusions. Forced drainage methods, like IFCO, to speed up the settlement traditionally rely on the availability of measured settlement and pore pressure data. This report shows that fairly accurate predictions for the settlement can be made when the project is still on the drawing board.

Limitations. The Isotach model has been tested especially on soft soils. A single application of the Isotach model on a situation with a layer of clayey sand does not remove the possible limitations of the model on that type of soil. A consultant should be aware of this lack of experience on less compressible soils, if he encounters that type of soils.

Availability. The calculations (postdictions) for settlement and pore pressures were done by means of the MSettle suite, version 6.7. The re-analysis of the Kp-CRS tests was partly performed by MCompress, version 1.0. The accessibility and user-friendliness of the software ensures a potentially broad distribution of the Isotach model in the geotechnical world, mainly consultancy firms.

Distribution. The Isotach model is being used by a limited group of companies. Information on which specific users is currently not available.

Costs. Application of the Isotach model and Kp-CRS tests involves costs, which are comparable to oedometer tests and e.g. a classical Koppejan calculation.

Risk of failure. Failure of the introduction of the Isotach model in settlement calculation depends on the availability of reliable data, especially soil parameters and information on loading. The Isotach model will be applied in those situations where an accurate prediction of settlement is necessary.

Time span. The Isotach model is available for the civil engineering society. DC partners have put effort in the introduction of the method. The Kp-CRS test, a specific method of laboratory testing, is not yet broadly integrated in the design process in the engineering companies. This will take its time. The method is likely to be introduced by Rijkswaterstaat, which can provide a technology push. Within a few years, maybe five years, it will be more broadly used, especially in those cases where settlements are expected to be high, or when several stages of loading and unloading will take place. Replacement of e.g. classical methods like Koppejan will take a decade.

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Societal Relevance of the research

Growing demand for mobility stresses the current infrastructure to the limits, especially in areas with a high economic productivity. Extending the existing infrastructure is the major concern of the current government. Limited financial resources and the growing liberalisation, aggravated by the increasing influence of Brussels, has driven the construction of roads and railroads towards privately owned or controlled parties. Receding financial and human resources have lead to solutions in the realm of Design, Construct and Maintain, DCM for short. Contractors are free to choose their own solution, as long as a number of vital demands, as imposed by the government, is fulfilled.

The benefits of design, construct and maintain is that the sum of all corresponding costs can be minimised. Since one organisation is responsible for all costs, the organisation has a good reason to minimise all costs, since benefits or disadvantages of a particular decision or solution will have impact in their own organisation. When responsibility for maintenance is relocated at the side of the contractor, he will require, next to the costs for the construction process, also an accurate estimate of the future costs. Future costs comprise mainly of maintenance as a result of settlement of the road. A further advantage of DCM is that a construction will be ready in a shorter period of time, i.e. the society can harvest the benefits of the investment as soon as possible. Nevertheless design, construct and maintain is not a panacea, since bureaucratic procedures substantially prolong the time between the conceptual design and the actual construction of a road.

Design and maintenance has a disadvantage as well. Roads have to be maintained for many decades. In the meanwhile a contractor can go bankrupt, leaving the government with the costs for the maintenance.

This report describes a method for the prediction of the long term settlement of a road, railroad or runway. This method enables the user to predict settlement, especially caused by creep in soft soils. These soils are mainly found in the Western part of the Netherlands, in the same area where economic activity is high. The results of this report will be used mainly by civil engineering companies or divisions that will advise a contractor on topics of settlement and deformation.

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Annexes
1 Introduction

1.1 General
When a runway is constructed on top of an undisturbed terrain, a substantial settlement is to be expected, especially when the subsoil is very soft. The reason is simple: the soil skeleton is compressible. Since soft soils tend to be poorly permeable, deformation is not instantaneous. A change in the volume of the soil means a change in the volume of the pore water as well. Since the water in the pores is hardly compressible, part of it should drain. The low permeability of the soil restricts the flow of the pore water. As a result the deformation is not instantaneous, but it takes quite some time. From a geotechnical point of view two processes play a role: consolidation and creep. Consolidation is mainly a dominant process during the initial phases of the construction. Creep produces long term settlement. The period in which creep is active, coincides with the period a road is to be maintained and serviced.

Several models have been devised in order to simulate consolidation and creep. These models predict settlement. Costs for the management and maintenance of roads and railroads are increasingly important, not in the least while private parties are increasingly kept responsible for the maintenance of roads. This aspect raised the demand for accurate predictions of settlements.

Since there is a market driven demand for a more accurate prediction of settlement than there used to be, a Delft Cluster project was initiated: "Samengestelde constructies". The aim of the project is to arrive at an integrated approach of soil and structure. The emphasis in this project is directed mainly towards the prediction of settlement caused by creep.

The main part of the project is the validation of the so called Isotach model or 'abc model'. This model is described by [den Haan 1994]. An implementation of this model can be found in the MSettle suite, which opens the Isotach model up for the geotechnical community. The afore mentioned models can boast on a large practical experience, which is certainly not the case for the Isotach model. In order to determine how accurately the model can predict long term settlement, calculations using the Isotach model are to be compared to settlements as measured in the field. The purpose of such a validation is to provide confidence in the model. This report describes is the validation of the Isotach or 'abc model' and the method to measure its input parameters in the laboratory.

1.2 Schiphol, 5th runway
Transport through Schiphol airport has grown to more than 40 million passengers yearly. The airport is under constant scrutiny of the government and environmental groups. By spreading take off and landing movements, the environmental load on neighbouring villages can be spread as well. Therefore the 5th runway, officially known as ‘18-36’ was commissioned. It will be in operation in February 2003. The runway is 3800 m long and stretches itself in North-South direction. Next to the runway there is a de-icing platform and a taxi lane. The runway is divided into several segments, designated by a character, like A and B. Each segment is divided into several sections, designated by a number. A section is a part of the runway, which from the point of view of IFCO, forms an entity.

The original plans were to set the 5th runway into operation in November 2002. Due to the short period of time between the initial phase of the construction and the anticipated moment the runway should be in operation, a forced method of consolidation was chosen: IFCO. The IFCO method comprises procedures during both construction and consultancy. The basic idea is to accelerate the consolidation by reduction of the pressure in vertical drains. Additionally a pressure difference over a foil, which seals a section from its surroundings, will act as a virtual surcharge.
The runway consists of a base layer of sand, roughly 0.5 m thick. The foundation consists of a layer of cemented granular material, 0.7 m thick. A layer of asphalt, 0.2 m thick, finishes the runway. During the period of forced consolidation, an equivalent load is present, consisting entirely of sand. After preloading and when the rate of settlement was sufficiently low, the excess sand was removed and the foundation layer and the asphalt were installed. The thickness of the equivalent layer of sand is 1.65 m. Part of the runway is below the original ground level. Therefore a trench of 0.6 m deep is dug into the ground. This trench is roughly perpendicular to the drainage trenches.

The water pressure in the drain is measured, as well as the pressure in the soil at −9.5, −10.25 and −11 m below the original ground level. Ground level itself is roughly −5m NAP. The settlement at the beginning of the measurements is set to zero. Unfortunately for the validation no measurements for settlement and pore pressures are available, before any changes in the original situation took place. Measurements are available in written form: reports. Digital data are not available. Two sections were chosen: A1 and A6. A boring (B01) was made near section A1. Samples were taken from this boring. On six of these samples $K_p$-CRS tests have been performed.

This report is written as part of article 2 sub 3 of DWW contract 2271. The financial resources for writing this report were made available by the market, i.e. Road and Hydraulic Engineering Division of the Directorate General of Public Works and Water Management.
2 Recent developments

The current report is a follow up of report [den Adel 2003]. For developments in the year 2002 see [den Adel 2003].

2.1 Preconsolidation stress

For the determination of the preconsolidation stress, usually the Casagrande method is followed. This method works with linear strain. The Isotach model however uses natural strain. Therefore it is more obvious to adapt the Casagrande method slightly and apply natural strain as well. Differences between the original and adapted Casagrande method are relatively small, of the order of 5%. In [den Adel 2003] it is shown that the agreement between measurement and calculation is slightly better when the adapted Casagrande method is applied for the preconsolidation stress.

2.2 Sill stress

The artificial sill stress was introduced [Sellmeijer 2002] in order to quench numerical oscillations. Analysis of the data of the “Waardse Alliantie” [den Haan 2002] has shown that the sill stress has an unacceptable large influence on the settlement. The sill stress has been replaced by the limit stress. In the calculations for this report limit stress was set to 0.

2.3 Strain dependent permeability

The Isotach model is likely to be applied when settlements -and thus strains- are expected to be high. When strains are high, the pore skeleton is deformed rather rigorously. The deformations have a large influence on the permeability of the soil. In K₀ tests a decrease in permeability is observed.

In the "Waardse Alliantie" report [den Haan 2002] it was shown that the assumption of a constant permeability is not good agreement with the measured settlement. In the MSettle suite a variable permeability was introduced:

\[ k(e) = k(e₀) \times 10^{C_{ke} \frac{e-e₀}{e₀}} \]

where \( e \) is the actual void ratio, \( e₀ \) is the initial void ratio, \( k(e₀) \) is the permeability at the initial void ratio and \( k(e) \) is the permeability at void ratio \( e \). The value of \( C_{ke} \) is determined experimentally in a K₀ test. Experimental proof of the dependence of permeability on strain is shown in Figure 2.1.
Figure 2.1 Observed increase in pore pressure as a function of strain

2.4 IFCO method

The IFCO method is a system of both mechanical engineering practice and consultancy. Consultancy relies on actual measurements of settlement and pore pressures. An appropriate monitoring system for the determination of settlement and pore pressures is part of the IFCO method. From an engineering point of view narrow trenches are dug into the subsoil, roughly perpendicular to the axis of the runway. In these trenches a drainage system is installed. The trenches are filled with permeable sand. The bottom of the trenches was kept at least at 2.5 m away from the impermeable layer of peat. Next to this impermeable layer a section of the runway is sealed from surrounding water and air pressure by means of bentonite shields and an impermeable foil. When in operation, the pumps reduce the water pressure in the drains to roughly 40 kPa less than the surrounding air. By lowering the water pressure in the drains, excess pore water will be forced to flow horizontally to the trenches. Since the distance between the trenches is roughly 3.5 m, the average distance to one of the vertical trenches is considerably shorter than to the ground level. Additionally by lowering the pressure in the drains the pressure difference between the subsoil and the surrounding air pressure will produce a virtual surcharge on the road, resulting in an accelerated consolidation.

The IFCO method deploys resources, like a pumping system, transducers and data loggers. Since these resources have a price, the most economic solution is to use forced drainage only during a limited amount of time. The consultancy is aimed at the deployment of the resources as long as its use has an advantage. That means that if the rate of deformations or the rate of decrease in pore pressures becomes too low, its more economic to let nature do its work and to progress the forced drainage to another section. Consultancy is aimed at detecting the turn over point at an early stage. Since consultancy uses differences in settlement and pore pressure in order to check whether the criterion to stop forced consolidation in a section, is met, there is no need for absolute data. This fact is rather unfortunate for our comparison between measurements and calculations: the values of the initial pore pressures and the initial height of the ground level is missing.
3 Experimental data

3.1 Selection of two sections for the study

According to people that were involved in the consultancy of the 5th runway, segment A is the most complete segment compared to other segments. Segment A is composed of 9 sections. Amongst them, only two sections have coherent experimental results for the measurement of the water pressures at different depths. That means that, at those locations, transducers do not oscillate. As a result the experimental results of those two sections are selected for the comparison with the MSettle predictions:

- Section A1 from km 3600 to 3800;
- Section A6 from km 2400 to 2600.

3.2 Geometry

The geometry consists of two parts: the soil, i.e. the foundation for the road and the road itself.

3.2.1 Soil

For the characterisation of the subsoil, three different sources are used:

- the geotechnical profile (map number 6, archives 175167) where three types of material are distinguished: sand, compressible soil and peat, see Table 3.1 and Table 3.2;
- the boring B01 made by GeoDelft at location km 3208, at 25 m of the west side of the runway, see Table 3.3;
- the boring B09 made by Tjaden at location km 2375, at 20 m of the centre of the runway, see Table 3.4.

<table>
<thead>
<tr>
<th>Material</th>
<th>depth [m…NAP]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>from</td>
</tr>
<tr>
<td>Ground level</td>
<td>-4.85</td>
</tr>
<tr>
<td>Compressible soil</td>
<td>-4.85</td>
</tr>
<tr>
<td>Sand</td>
<td>-6.10</td>
</tr>
<tr>
<td>Compressible soil</td>
<td>-6.80</td>
</tr>
<tr>
<td>Sand</td>
<td>-8.50</td>
</tr>
<tr>
<td>Compressible soil</td>
<td>-9.90</td>
</tr>
<tr>
<td>Peat</td>
<td>-12.10</td>
</tr>
<tr>
<td>Pleistocene sand</td>
<td>-12.70</td>
</tr>
</tbody>
</table>

*Table 3.1*  
Section A1 - The layers in the soil at km 3700 according to the geotechnical profile
<table>
<thead>
<tr>
<th>Material</th>
<th>depth [m...NAP]</th>
</tr>
</thead>
<tbody>
<tr>
<td>from</td>
<td>till</td>
</tr>
<tr>
<td>Ground level</td>
<td>-4.90</td>
</tr>
<tr>
<td>Compressible soil</td>
<td>-4.90 -6</td>
</tr>
<tr>
<td>Sand</td>
<td>-6 -8.50</td>
</tr>
<tr>
<td>Compressible soil</td>
<td>-8.50 -12.60</td>
</tr>
<tr>
<td>Peat</td>
<td>-12.60 -13</td>
</tr>
<tr>
<td>Pleistocene sand</td>
<td>-13</td>
</tr>
</tbody>
</table>

Table 3.2  Section A6 - The layers in the soil at km 2500 according to the geotechnical profile

<table>
<thead>
<tr>
<th>Material</th>
<th>Nr. layer</th>
<th>Sample</th>
<th>depth [m...NAP ]</th>
<th>( \gamma_{\text{wet}} )</th>
<th>( \gamma_{\text{dry}} )</th>
<th>Water content</th>
</tr>
</thead>
<tbody>
<tr>
<td>from</td>
<td>till</td>
<td>[kN/m^3]</td>
<td>[%]</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ground level</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand, moderately silty</td>
<td>1</td>
<td></td>
<td>-5.05</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay, moderately silty, weakly organic</td>
<td>2</td>
<td>53</td>
<td>-5.35 -5.65</td>
<td>18.7</td>
<td>14.5</td>
<td>29.1</td>
</tr>
<tr>
<td>Clay, very silty</td>
<td>3</td>
<td></td>
<td>-5.65 -6.10</td>
<td>16.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand, moderately silty</td>
<td>4</td>
<td></td>
<td>-6.10 -6.90</td>
<td>17.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand, very silty, chunks of clay, broken shells</td>
<td>5</td>
<td></td>
<td>-6.90 -8.43</td>
<td>18.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand, clayey, broken shells</td>
<td>6 56a</td>
<td></td>
<td>-8.43 -9.67</td>
<td>16.0</td>
<td>10.0</td>
<td>59.8</td>
</tr>
<tr>
<td>Clay, very silty, broken shells</td>
<td>7 57</td>
<td></td>
<td>-9.67 -10.05</td>
<td>15.5</td>
<td>9.0</td>
<td>71.7</td>
</tr>
<tr>
<td>Clay, moderately silty, weakly organic, remains of plants</td>
<td>8</td>
<td>58</td>
<td>-10.05 -12.10</td>
<td>14.4</td>
<td>7.8</td>
<td>83.8</td>
</tr>
<tr>
<td>Clay, moderately silty</td>
<td>9 59b</td>
<td></td>
<td>-12.10 -12.48</td>
<td>13.3</td>
<td>5.9</td>
<td>126.7</td>
</tr>
<tr>
<td>Peat, poor on minerals</td>
<td>10 60</td>
<td></td>
<td>-12.48 -12.73</td>
<td>10.5</td>
<td>2.5</td>
<td>328.0</td>
</tr>
<tr>
<td>Pleistocene sand</td>
<td></td>
<td></td>
<td>-12.73</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.3  Boring GeoDelft B01 km 3208 - The layers in the soil
<table>
<thead>
<tr>
<th>Material</th>
<th>Nr. layer</th>
<th>Sample</th>
<th>depth [m...NAP ]</th>
<th>$\gamma_{\text{wet}}$</th>
<th>$\gamma_{\text{dry}}$</th>
<th>Water content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground level</td>
<td></td>
<td></td>
<td>-4.84</td>
<td>-5.44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay, compact, dark/brown, with fine sand, with remains of rubbish</td>
<td>1</td>
<td></td>
<td>-4.84</td>
<td>-5.44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay, compact, dark/brown with remains of shells</td>
<td>2</td>
<td></td>
<td>-5.44</td>
<td>-5.94</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand, fine, brown, slightly clayey, with remains of iron</td>
<td>3</td>
<td></td>
<td>-5.94</td>
<td>-6.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand, moderately fine, grey, slightly clayey, with remains of shells and iron</td>
<td>4</td>
<td>3</td>
<td>-6.04</td>
<td>-6.84</td>
<td>19.5</td>
<td>15.7</td>
</tr>
<tr>
<td>Sand, moderately fine, grey, with remains of shells</td>
<td>5</td>
<td></td>
<td>-6.84</td>
<td>-7.84</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay, moderately soft, grey</td>
<td>6</td>
<td>6</td>
<td>-7.84</td>
<td>-9.64</td>
<td>15.7</td>
<td>9.6</td>
</tr>
<tr>
<td>Clay, moderately soft, grey, with remains of shells</td>
<td>7</td>
<td></td>
<td>-9.64</td>
<td>-9.84</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay, moderately soft, brown, strongly peaty</td>
<td>8</td>
<td>9</td>
<td>-9.84</td>
<td>-9.94</td>
<td>15.7</td>
<td>6.7</td>
</tr>
<tr>
<td>Clay, moderately soft, grey/brown, moderately peaty</td>
<td>9</td>
<td>10</td>
<td>-9.94</td>
<td>-10.84</td>
<td>13.9</td>
<td>7</td>
</tr>
<tr>
<td>Clay, moderately compact, grey, moderately peaty</td>
<td>10</td>
<td></td>
<td>-10.84</td>
<td>-11.44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay, moderately compact, grey, slightly peaty</td>
<td>11</td>
<td></td>
<td>-11.44</td>
<td>-12.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay, moderately compact, grey</td>
<td>12</td>
<td></td>
<td>-12.04</td>
<td>-12.44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Peat, dark/brown</td>
<td>13</td>
<td>13</td>
<td>-12.44</td>
<td>-12.64</td>
<td>13.6</td>
<td>5.9</td>
</tr>
<tr>
<td>Pleistocene sand</td>
<td>14</td>
<td></td>
<td>-12.64</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.4 Boring Tjaden B09 km 2375 - The layers in the soil

Borings B01 (GeoDelft) and B09 (Tjaden) are close to respectively sections A1 and A6, as shown in Figure 3.1. Boring B01 is made after the runway was constructed, B09 before its construction. Therefore boring B01 had to be positioned next to the runway.
Figure 3.1 Location of the borings B01 and B09 compared to sections A1 and A6

For both sections, the sand and peat layers in the geotechnical profile correspond respectively with sample 3 of boring B09 and sample 60 of the boring B01. The compressible soil at the top of the geotechnical profile corresponds with sample 53 of boring B01.

For section A6, the compressible layer between -8.5 m and -12.6 m NAP is divided into four clusters using the same set of layers of boring B09 and the following correspondence with boring B01:
- layer 6 of boring B09 corresponds with sample 56a of boring B01, since the characteristics are very close;
- layer 7 of boring B09 corresponds with sample 57 of boring B01, since both mention shells;
- layers 8, 9, 10 and 11 correspond with sample 58 of boring B01, since it describes a clay slightly or moderately peaty and the characteristics are very close;
- layer 12 of boring B09 corresponds with sample 59b of boring B01, since both describe a clay moderately silty.

For section A1, the compressible layer between -6.8 m and -8.50 m NAP correspond with sample 56a since it is a sand with chunks of clay. And the compressible layer between -9.90 m and -12.10 m NAP is divided into three clusters using the same set of layers of boring B01.

The set of layers for sections A1 and A6 is shown in Table 3.5 and Table 3.6. A graph of the geometry is shown in annexes G_A1 for section A1 and G_A6 for section A6.
Table 3.5  Section A1 - The layers in the soil

<table>
<thead>
<tr>
<th>Material</th>
<th>depth [m...NAP]</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground level</td>
<td>-4.85</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay (B01 - sample 53)</td>
<td>-4.85</td>
<td>-6.10</td>
<td></td>
</tr>
<tr>
<td>Sand (B09 - sample 3)</td>
<td>-6.10</td>
<td>-6.80</td>
<td></td>
</tr>
<tr>
<td>Sand clayey (B01 - sample 56a)</td>
<td>-6.80</td>
<td>-8.50</td>
<td></td>
</tr>
<tr>
<td>Sand (B09 - sample 3)</td>
<td>-8.50</td>
<td>-9.90</td>
<td></td>
</tr>
<tr>
<td>Clay (B01 - sample 57)</td>
<td>-9.90</td>
<td>-10.20</td>
<td></td>
</tr>
<tr>
<td>Clay (B01 - sample 58)</td>
<td>-10.20</td>
<td>-11.80</td>
<td></td>
</tr>
<tr>
<td>Clay (B01 - sample 59b)</td>
<td>-11.80</td>
<td>-12.10</td>
<td></td>
</tr>
<tr>
<td>Peat (B01 - sample 60)</td>
<td>-12.10</td>
<td>-12.70</td>
<td></td>
</tr>
<tr>
<td>Pleistocene sand</td>
<td>-12.70</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.6  Section A6 - The layers in the soil

<table>
<thead>
<tr>
<th>Material</th>
<th>depth [m...NAP]</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground level</td>
<td>-4.90</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay (B01 - sample 53)</td>
<td>-4.90</td>
<td>-6.00</td>
<td></td>
</tr>
<tr>
<td>Sand (B09 – sample 3)</td>
<td>-6.00</td>
<td>-8.50</td>
<td></td>
</tr>
<tr>
<td>Sand clayey (B01 - sample 56a)</td>
<td>-8.50</td>
<td>-10.10</td>
<td></td>
</tr>
<tr>
<td>Clay (B01 - sample 57)</td>
<td>-10.10</td>
<td>-10.30</td>
<td></td>
</tr>
<tr>
<td>Clay (B01 - sample 58)</td>
<td>-10.30</td>
<td>-10.25</td>
<td></td>
</tr>
<tr>
<td>Clay (B01 - sample 59b)</td>
<td>-10.25</td>
<td>-12.6</td>
<td></td>
</tr>
<tr>
<td>Peat (B01 - sample 60)</td>
<td>-12.6</td>
<td>-13</td>
<td></td>
</tr>
<tr>
<td>Pleistocene sand</td>
<td>-13</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The interface between adjacent layers in a cross section of the sub soil is assumed to be horizontal.

3.2.2 Runway
From a civil engineering point of view a road at a height of about 1.25 meters above ground level had to be constructed. The runway was constructed in two phases. The IFCO pre-loading method with horizontal drainage was applied in the first phase, in order to minimise the time needed for a safe construction of the runway. Then in the second phase conventional techniques were applied for the construction of the actual runway.

In this study, only the first phase, i.e. the pre-loading that uses the IFCO method, is modelled with different steps:
1. Excavation of the subsoil, providing space for the foundation layer, until 0.40 m below the ground level;
2. Installation of the IFCO drainage system;
3. Filling of the foundation trench (height of 0.50 m) with sand;
4. Installation of the foil;
5. Raising the road with 0.8 m of sand.

The geometry of the successive steps of the construction of the road is presented in map number 97 (archive number 175258), and shown in Figure 3.2.
Figure 3.2  The geometry in steps for both sections A1 and A6

The log of section A1 (see Table 3.7) is quite consistent and complete; this is not the case for section A6. Unfortunately our copy of the contractors log runs until May 14th. On that day section A1 is finished and ready for switching on the drainage system. Section A6 is not yet ready and unfortunately exact data on the date trenches were dug, foil was added etc. is missing. Efforts to obtain the missing information were in vain. The contractor no longer could provide logs with the sufficient level of details as for section A1. To fill in the missing information for section A6, the same differences in time between consecutive steps as for section A1 are used (see Table 3.8).
Table 3.7  The steps in the construction of the new runway according to the log

<table>
<thead>
<tr>
<th>Section A1</th>
<th>Date</th>
<th>Action</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>2001-03-19</td>
<td>Excavate</td>
<td></td>
<td>Digging the trench (km 3800 to 3700)</td>
</tr>
<tr>
<td>2001-03-27</td>
<td>Excavate</td>
<td></td>
<td>Digging the trench (km 3700 to 3600)</td>
</tr>
<tr>
<td>2001-03-28</td>
<td>Heighten</td>
<td></td>
<td>Sand in the trench (km 3800 to 3600) – height 0.75 m</td>
</tr>
<tr>
<td>2001-04-11</td>
<td>Profile</td>
<td></td>
<td>Profiling the sand for installation of foil (km 3800 to 3700)</td>
</tr>
<tr>
<td>2001-04-23</td>
<td>Profile</td>
<td></td>
<td>Profiling the sand for installation of foil (km 3700 to 3600)</td>
</tr>
<tr>
<td>2001-04-23</td>
<td>Install foil</td>
<td></td>
<td>Installation of the foil (km 3800 to 3700)</td>
</tr>
<tr>
<td>2001-04-24</td>
<td>Install foil</td>
<td></td>
<td>Installation of the foil (km 3700 to 3600)</td>
</tr>
<tr>
<td>2001-04-24</td>
<td>Heighten</td>
<td></td>
<td>Sand (km 3700 to 3600) – height 0.90 m</td>
</tr>
<tr>
<td>2001-05-07</td>
<td>Heighten</td>
<td></td>
<td>Sand (km 3800 to 3700) – height 0.90 m</td>
</tr>
<tr>
<td>2001-05-07</td>
<td>Heighten</td>
<td></td>
<td>Pre-loading of section A1 was finished at week 18</td>
</tr>
<tr>
<td>2001-05-06</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2001-05-07</td>
<td>Excavate</td>
<td></td>
<td>Continuation of digging the trench (km 2600 to 2500)</td>
</tr>
<tr>
<td>2001-05-14</td>
<td>Excavate</td>
<td></td>
<td>Digging the trench (km 2400 to 2300)</td>
</tr>
<tr>
<td>2001-06-11</td>
<td>Heighten</td>
<td></td>
<td>Pre-loading of section A6 was finished at week 24</td>
</tr>
<tr>
<td>2001-05-17</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The building steps as used in the calculations, are explained in Table 3.8.

Table 3.8  The steps in the construction of the new runway, as used in the calculations

<table>
<thead>
<tr>
<th>Day</th>
<th>Date</th>
<th>Action</th>
<th>( \gamma ) ([\text{kN/m}^3])</th>
<th>Height From ([\text{m}])</th>
<th>Height Till ([\text{m}])</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2001-03-23</td>
<td>Excavate</td>
<td>-18.7</td>
<td>0</td>
<td>-0.40</td>
<td>Digging</td>
</tr>
<tr>
<td>12</td>
<td>2001-04-04</td>
<td>Heighten</td>
<td>20</td>
<td>-0.40</td>
<td>0.45</td>
<td>0.85 m</td>
</tr>
<tr>
<td>19</td>
<td>2001-04-11</td>
<td>Pumping</td>
<td>0.45</td>
<td>0.45</td>
<td>1.25</td>
<td>Start of pumping</td>
</tr>
<tr>
<td>39</td>
<td>2001-05-01</td>
<td>Heighten</td>
<td>20</td>
<td>0.45</td>
<td>1.25</td>
<td>0.80 m</td>
</tr>
<tr>
<td>165</td>
<td>2001-09-04</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>End of pumping</td>
</tr>
<tr>
<td>0</td>
<td>2001-05-03</td>
<td>Excavate</td>
<td>-18.7</td>
<td>0</td>
<td>-0.40</td>
<td>Digging</td>
</tr>
<tr>
<td>12</td>
<td>2001-05-15</td>
<td>Heighten</td>
<td>20</td>
<td>-0.40</td>
<td>0.45</td>
<td>0.85 m</td>
</tr>
<tr>
<td>39</td>
<td>2001-06-11</td>
<td>Heighten</td>
<td>20</td>
<td>0.45</td>
<td>1.25</td>
<td>0.80 m</td>
</tr>
<tr>
<td>43</td>
<td>2001-06-15</td>
<td>Pumping</td>
<td>0.45</td>
<td>0.45</td>
<td></td>
<td>Start of pumping</td>
</tr>
<tr>
<td>175</td>
<td>2001-10-25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>End of pumping</td>
</tr>
</tbody>
</table>

3.2.3 Vertical drainage

The IFCO drainage system consists of a series of drainage trenches, filled with sand. At the bottom of the trench a drainage pipe is placed. The pressure in this pipe is reduced. The end position of the drains is 2.5 m above the Pleistocene sand. The distance between two drains is 3.5 m and the diameter of the drains is roughly 0.025 m.

The experimental difference in pressure in the drain, \( \Delta P_{\text{meas}} \), is measured. The value is given relative to the atmospheric pressure:
\[ \Delta P_{\text{meas}} = P_{\text{atm}} - P_{\text{meas}} \]  

(3.1)

where \( P_{\text{meas}} \) is the pressure given by the sensor in the drain

\( P_{\text{atm}} \) is the atmospheric pressure.

A pressure difference of \( \Delta P_{\text{meas}} = 30 \, \text{kPa} \) corresponds to a decrease of the water level in the drainage trench of approximately 3 m. In contrary, a difference in pressure of \(-30 \, \text{kPa}\) corresponds to an increase of 3 m of the water level.

The input value for calculations with MSettle is the applied pressure \( P_{\text{pipe}} \) in the drainage pipe. So, the pressure difference in the drain \( \Delta P_{\text{meas}} \) must be substracted from the hydrostatic pressure at the depth where the pipe is located in order to get the absolute pressure. This leads to:

\[ P_{\text{pipe}} = (z_{\text{water}} - z_{\text{pipe}}) \gamma_w - \Delta P_{\text{meas}} \]  

(3.2)

where \( z_{\text{pipe}} \) is the vertical co-ordinate of the drainage tube

\( z_{\text{water}} \) is the vertical co-ordinate of the phreatic line

\( \gamma_w \) is the water weight.

The values of the pressure in the pipe, used in MSettle, for both sections are shown in Figure 3.4 and Figure 3.5.

The pressure at the top of the drain, \( \Delta P_{\text{air}} \), - in the sand above the freatic line - was not measured. So two extreme cases are assumed.

In case of an ideal working of the sealing foil at the top of the drain, the air pressure \( P_{\text{air}} \) is equal to the pressure in the pipe:

\[ \Delta P_{\text{air}} = P_{\text{atm}} - P_{\text{air}} = P_{\text{atm}} - P_{\text{pipe}} \]  

(3.3)

In case of a malfunction of the sealing foil, the air pressure is equal to the atmospheric pressure:

\[ \Delta P_{\text{air}} = P_{\text{atm}} - P_{\text{air}} = 0 \]  

(3.4)
Figure 3.4  Section A1 – Pressure in the drain pipe as a function of time

Figure 3.5  Section A6 – Pressure in the drain pipe as a function of time
3.2.4 Phreatic line
According to the report 95275-2 “Advies met betrekking tot het bouwrijp maken ten behoeve van de 5de baan op de luchthaven Schiphol” (paragraph 3.0, page 2), the phreatic line of borings 5 (km 3300) and 11 (km 2100) located in the North is at -6.50 m NAP and the phreatic line of borings 17 (km 900) and 46 (km 0) located in the South is at -6.00 m NAP. By extrapolation (see Figure 3.6), the phreatic line for sections A1 and A6 is set equal to -6.5 m NAP.

![Figure 3.6 The phreatic line level at different locations](image)

3.2.5 Piezometric line for the Pleistocene sand
According to the report CO-374820/44 “Alternatief Voorontwerp 5e baan Schiphol schuimbeton toepassing” (paragraph 8.2.3, page 40), the total head in the Pleistocene sand varies between -4.2 and -4.5 m NAP at the Northern end of the runway and between -5.5 and 5.9 m NAP at the Southern end. Since no other data between both ends are available, the value of the total head in the Pleistocene sand is obtained by linear interpolation. The resulting heads in the Pleistocene sand is -4.4m and -4.8 m NAP respectively for sections A1 and A6, see Figure 3.8.

Section A1 is situated near RD coordinates 109020, 486025; section A6 near RD coordinates 108950, 485050.
A query in TNO-NITG’s DINO database revealed that data at RD 108200, 487300 is available: filter 25CP0376_01, see Figure 3.7. Note that this filter is approximately some 5 km away from the runway. It is a well known fact that the hydraulic heads in the Haarlemmermeer are subjected to local variations. Therefore it is not possible to make an accurate prediction for the hydraulic head in sections A1 and A6, based on the available DINO data. Figure 3.7 nevertheless confirms the assumptions, the margins included, made for the head in the pleistocene sand.
Figure 3.7  The head in the pleistocene sand at RD 108200, 487300

Figure 3.8  The hydraulic head in the Pleistocene sand at different locations.
3.3 Settlement

For each section, the settlements are measured in 12 different locations, 12.5 m beside the centre of the road. For the comparison with MSettle predictions, an average value is used, with an uncertainty of +/- 0.015 m for section A1 and +/- 0.020 m for section A6 (see Figure 3.9 and Figure 3.10).

According to the report number 401960.27 "Eindrapportage monitoring project bouwrijp maken 5de baan Schiphol", the settlement after 30 years was calculated to be 0.25 m and 0.37 m respectively for sections A1 and A6.

---

*Figure 3.9  Section A1 – Average measured settlement*
3.4 Pore pressures

Pore pressures have been measured at different locations along the centre of the road and at three different depths: -9.5 m, -10.25 m and -11 m NAP (see Figure 3.11 and Figure 3.12). Sensors are located in the middle between two drains.
Figure 3.11  Section A1 - Water pressure measured by the pore pressure transducers

Figure 3.12  Section A6 - Water pressure measured by the pore pressure transducers
3.5 Interpretation of the $K_0$-CRS tests

Six samples from the boring at km 3208 were tested.

3.5.1 $K_0$-CRS test

The $K_0$-CRS test has been developed by GeoDelft to determine the Isotach parameters $a$, $b$ and $c$. This test is performed at a constant rate of strain. During the test, the displacement, the stresses at the top and at the bottom of the sample and the water pressure are measured.

Two phases are included in the test:
- an unloading/reloading phase, during which the Isotach swelling index $a$ can be determined;
- a relaxation phase, during which the Isotach compression coefficient $c$ can be determined.

Parameter $b$ is the final slope of the curve $\sigma_v' = e^H$ (see Figure 3.13).

![Diagram showing interpretation of the result of the $K_0$-CRS test for the determination of $a$ and $b$.]

The evaluation of the $K_0$-CRS test data is described in more detail in reports [den Haan, 2001] and [den Adel, 2002].

3.5.2 Isotach parameters: $a$, $b$ and $c$

In the spring of 2002 the program MCompress has been developed. Its aim is to evaluate the results of oedometer tests and $K_0$ tests. The data of the $K_0$ tests have been analysed manually in a spreadsheet and by MCompress. The results are shown in Table 3.9 and Table 3.10.
<table>
<thead>
<tr>
<th>Material</th>
<th>a</th>
<th>b</th>
<th>c</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay (sample 53)</td>
<td>3.137E-03</td>
<td>3.201E-02</td>
<td>1.243E-03</td>
</tr>
<tr>
<td>Sand (sample 56a)</td>
<td>8.523E-03</td>
<td>1.035E-01</td>
<td>5.417E-03</td>
</tr>
<tr>
<td>Clay (sample 57)</td>
<td>9.008E-03</td>
<td>1.513E-01</td>
<td>8.320E-03</td>
</tr>
<tr>
<td>Clay (sample 58)</td>
<td>1.337E-02</td>
<td>1.704E-01</td>
<td>9.562E-03</td>
</tr>
<tr>
<td>Clay (sample 59b)</td>
<td>1.427E-02</td>
<td>2.321E-01</td>
<td>1.238E-02</td>
</tr>
<tr>
<td>Peat (sample 60)</td>
<td>2.106E-02</td>
<td>2.979E-01</td>
<td>1.454E-02</td>
</tr>
</tbody>
</table>

Table 3.9  Values of a, b and c as determined with MCompress

<table>
<thead>
<tr>
<th>Material</th>
<th>b</th>
<th>c</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay (sample 53)</td>
<td>4.518E-02</td>
<td>1.730E-03</td>
</tr>
<tr>
<td>Sand (sample 56a)</td>
<td>1.197E-01</td>
<td>2.470E-03</td>
</tr>
<tr>
<td>Clay (sample 57)</td>
<td>1.795E-01</td>
<td>1.010E-02</td>
</tr>
<tr>
<td>Clay (sample 58)</td>
<td>1.825E-01</td>
<td>1.090E-02</td>
</tr>
<tr>
<td>Clay (sample 59b)</td>
<td>2.389E-01</td>
<td>1.490E-02</td>
</tr>
<tr>
<td>Peat (sample 60)</td>
<td>3.225E-01</td>
<td>1.870E-02</td>
</tr>
</tbody>
</table>

Table 3.10  Values of b and c as determined manually

When we compare the b value from MCompress with the one determined manually, there is a significant difference between MCompress and the manual method. MCompress uses the last part of the test (after the relaxation phase, i.e. the horizontal indentation at 0.17 of natural strain in Figure 3.14) whereas in the manual method we use the part of the test between the unloading/reloading phase and the relaxation phase. This method is similar to one as is used in [den Adel 2003].

![Figure 3.14](image-url)  Determination of parameter b manually and with MCompress

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A possible cause for the buckle in the virgin section of the stress strain relation may be contributed to the fact that the test has been performed with a constant rate of linear strain. According to theory the virgin line in Figure 3.14 would have been a straight line if the rate of natural strain would have been constant. The effect of the difference between linear and natural strain rates is explained in section 8. The numerical influence is shown to be negligible.

In the following calculations we will use for parameter a the results of MCompress whereas for parameters b and c we will use the values determined manually.

In both sections part of the sub soil consists of sand, see Table 3.5 and Table 3.6. Since it is rather impractical to subject a sand sample to a K0-CRS test, the values of a and b had to be calculated from the oedometer test data, as provided by Tjaden. Since creep in sand is assumed to be negligible compared to creep in clay and peat, the value of c is assumed to be small: 10⁻⁶. For a, a value of 0.01108 is found; for b 0.04472.

### 3.5.3 Pre-overburden pressure

The pre-overburden pressure (POP) is defined as:

\[
POF = p_{e} - \sigma'_{v}
\]  

(3.4)

where \(p_{e}\) is the preconsolidation stress and \(\sigma'_{v}\) is the actual effective vertical stress.

The \(p_{e}\) value is determined in the spreadsheet using the Casagrande method with natural strain, as shown in Figure 3.15.

![Figure 3.15: Determination of the preconsolidation stress \(p_{e}\) manually](image)

---

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The values as used in the calculations in section 4, are shown in Table 3.11:

<table>
<thead>
<tr>
<th>Material</th>
<th>$\sigma'_v$ [kPa]</th>
<th>$p_K$ [kPa]</th>
<th>POP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay (sample 53)</td>
<td>9.2</td>
<td>99.4</td>
<td>90.2</td>
</tr>
<tr>
<td>Sand (sample 56a)</td>
<td>36.9</td>
<td>66.5</td>
<td>29.6</td>
</tr>
<tr>
<td>Clay (sample 57)</td>
<td>40.8</td>
<td>98.3</td>
<td>57.5</td>
</tr>
<tr>
<td>Clay (sample 58)</td>
<td>42.4</td>
<td>83.9</td>
<td>41.5</td>
</tr>
<tr>
<td>Clay (sample 59b)</td>
<td>44.4</td>
<td>123.4</td>
<td>79.0</td>
</tr>
<tr>
<td>Peat (sample 60)</td>
<td>44.3</td>
<td>189.3</td>
<td>145.0</td>
</tr>
</tbody>
</table>

*Table 3.11 The effective stress, preconsolidation stress and pre-overburden pressure.*

According to Figure 3.16, the value of the pre-overburden pressure in the top layer (sample 53) seems too high compared to the others layers. However, as the relative settlement of this layer doesn’t have a large influence on the total settlement, this POP value will be used in the calculations. For sample 3 of boring B09, the pre-consolidation stress was determined using a compression test. This method uses only a few points compared to the $K_0$-CRS test. As a result, the difference in the value of the preconsolidation stress between oedometer and $K_0$-CRS test can reach 30%, as shown in [den Adel, 2003]. For this reason, the POP value of sample 3 of boring B09 is extrapolated from the POP values of the other samples. The result is about 20 kPa.

![Figure 3.16 Distribution of the pre-overburden pressure along the boring B01](image)

**Figure 3.16 Distribution of the pre-overburden pressure along the boring B01**

### 3.5.4 Permeability

In 2002, a strain dependent permeability was introduced in MSettle. This feature is not yet present in MCompress. The analysis of the measured data was done by means of a spreadsheet. For a typical graph see Figure 3.17.
The results of the analysis for all samples, comparable to Figure 3.17, are shown in Table 3.12.

<table>
<thead>
<tr>
<th>Material</th>
<th>Vertical permeability $k(e_0)$ [m/d]</th>
<th>$\gamma_{wet}$ [kN/m$^3$]</th>
<th>$\gamma_{dry}$ [kN/m$^3$]</th>
<th>Porosity [-]</th>
<th>Void ratio $e_0$ [-]</th>
<th>$C_{ke}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay (sample 53)</td>
<td>4.317x10$^{-4}$</td>
<td>18.7</td>
<td>14.5</td>
<td>0.43</td>
<td>0.754</td>
<td>0.144</td>
</tr>
<tr>
<td>Sand (sample 56a)</td>
<td>2.320x10$^{-4}$</td>
<td>16.0</td>
<td>10.0</td>
<td>0.61</td>
<td>1.564</td>
<td>0.927</td>
</tr>
<tr>
<td>Clay (sample 57)</td>
<td>6.348x10$^{-5}$</td>
<td>15.5</td>
<td>9.0</td>
<td>0.66</td>
<td>1.941</td>
<td>1.160</td>
</tr>
<tr>
<td>Clay (sample 58)</td>
<td>6.259x10$^{-4}$</td>
<td>14.4</td>
<td>7.8</td>
<td>0.67</td>
<td>2.030</td>
<td>0.646</td>
</tr>
<tr>
<td>Clay (sample 59b)</td>
<td>7.203x10$^{-5}$</td>
<td>13.3</td>
<td>5.8</td>
<td>0.77</td>
<td>3.167</td>
<td>1.292</td>
</tr>
<tr>
<td>Peat (sample 60)</td>
<td>8.873x10$^{-4}$</td>
<td>10.5</td>
<td>2.5</td>
<td>0.82</td>
<td>4.556</td>
<td>1.173</td>
</tr>
</tbody>
</table>

Table 3.12 Determination of strain dependent permeability parameters (manually)

For the meaning of $k(e_0)$, $e_0$ and $C_{ke}$ is referred to equation (2.1). The value of $e_0$ is the initial value of the void ratio, as observed in the test.

![Graph showing determination of the strain dependent permeability](image)

**Figure 3.17** Determination of the strain dependent permeability

The horizontal permeability of clay and sand layers is set equal to the vertical permeability, while for the peat layer it is 4 times the vertical permeability.

In Figure 3.17 several strings of points can be distinguished. The mainly vertically oriented strings originate from unloading and reloading phases in the experiment. The rather horizontally oriented strings are obtained during continuous sections in the loading scheme.
For sample 3 of boring B09 ("sand"), a permeability of $1.3 \times 10^{-4}$ m/day was found using the Taylor method. In [den Adel, 2002], it is shown that the permeability determined from the Taylor method is about 50% more than the permeability determined using the K$_{cr}$-CRS results. For this reason, the permeability of sample 3 of boring B09 is set equal to $6.5 \times 10^{-5}$ m/day.
4 Calculations

4.1 The cases

Experimental data, as presented in section 3, has been used in the calculations. The Isotach model has been programmed in the MSettle suite. Version 6.7 has been used. Several cases have been calculated, listed in Table 4.1.

<table>
<thead>
<tr>
<th>Case</th>
<th>Air pressure in the drain</th>
<th>( \gamma ) [kN/m(^3)]</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>( P_{\text{pipe}} )</td>
<td>20</td>
<td>Basic situation, best guess</td>
</tr>
<tr>
<td>2</td>
<td>( P_{\text{atmospheric}} )</td>
<td>20</td>
<td>Leakage of the sealing system</td>
</tr>
<tr>
<td>3</td>
<td>( P_{\text{pipe}} )</td>
<td>20 (above the foil) 18 (below the foil)</td>
<td>Sand under the foil is slightly drier</td>
</tr>
<tr>
<td>4</td>
<td>( P_{\text{pipe}} )</td>
<td>20</td>
<td>Low value of the total head in the Pleistocene sand (-5.9 m NAP)</td>
</tr>
<tr>
<td>5</td>
<td>( P_{\text{pipe}} )</td>
<td>20</td>
<td>High value of the total head in the Pleistocene sand (-4.2 m NAP)</td>
</tr>
<tr>
<td>6</td>
<td>( P_{\text{pipe}} )</td>
<td>20</td>
<td>Phreatic line level at -6 m NAP</td>
</tr>
<tr>
<td>7</td>
<td>( P_{\text{pipe}} )</td>
<td>20</td>
<td>The second load starts at ( t = 17 ) (only for section A6)</td>
</tr>
</tbody>
</table>

*Table 4.1 Cases*

These cases have been introduced to visualise the influence that specific uncertainties in the parameters and in boundary conditions have on the calculated settlement.

Case 1 assumes an ideal working of the sealing of a section, specially the foil. This implies that the air pressure in the sand of the embankment is equal to the pressure in the drainage pipe.

On the contrary, case 2 assumes a malfunction of sealing, so the air pressure in the sand of the embankment is equal to the atmospheric pressure.

Case 3 illustrates the effect of partial saturation of the sand in the embankment, due to withdrawal of water.

Cases 4 and 5 illustrate the uncertainty in the value of the total head in the Pleistocene sand whereas case 6 illustrates the uncertainty on the value of the phreatic line.

Case 7 (for section A6 only) illustrates the uncertainty in the date of the construction of the top part of the runway.

The results of the calculations for the settlement are shown in Figure 4.1 till Figure 4.4. Cases are grouped into two sets. In each set the best guess case 1 is present. Set 1 contains cases 4, 5 and 6, all related to the uncertainty in head. Set 2 contains cases 2 and 3, related to mechanical loading.

The results of the calculations for the settlement after 30 years are shown in Table 4.2 and Table 4.3. For all cases, the final settlement predicted by MSettle is compared to the value calculated by IFCO (see paragraph 3.3). The relative difference is the difference between the Isotach prediction and the IFCO prediction, compared to the IFCO value.
<table>
<thead>
<tr>
<th>Case</th>
<th>Settlements after 10 000 days [m]</th>
<th>Relative difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>at the centre</td>
<td>at 12.5 m of the centre</td>
</tr>
<tr>
<td>1</td>
<td>0.205</td>
<td>0.186</td>
</tr>
<tr>
<td>2</td>
<td>0.185</td>
<td>0.164</td>
</tr>
<tr>
<td>3</td>
<td>0.189</td>
<td>0.169</td>
</tr>
<tr>
<td>4</td>
<td>0.172</td>
<td>0.154</td>
</tr>
<tr>
<td>5</td>
<td>0.210</td>
<td>0.191</td>
</tr>
<tr>
<td>6</td>
<td>0.221</td>
<td>0.203</td>
</tr>
</tbody>
</table>

*Table 4.2  Section A1 – Settlements after 30 years. IFCO’s prediction is 0.25 m*

<table>
<thead>
<tr>
<th>Case</th>
<th>Settlements after 10 000 days</th>
<th>Relative difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>at the centre</td>
<td>at 12.5 m of the centre</td>
</tr>
<tr>
<td>1</td>
<td>0.307</td>
<td>0.290</td>
</tr>
<tr>
<td>2</td>
<td>0.263</td>
<td>0.244</td>
</tr>
<tr>
<td>3</td>
<td>0.291</td>
<td>0.273</td>
</tr>
<tr>
<td>4</td>
<td>0.264</td>
<td>0.247</td>
</tr>
<tr>
<td>5</td>
<td>0.329</td>
<td>0.315</td>
</tr>
<tr>
<td>6</td>
<td>0.318</td>
<td>0.301</td>
</tr>
<tr>
<td>7</td>
<td>0.308</td>
<td>0.290</td>
</tr>
</tbody>
</table>

*Table 4.3  Section A6 – Settlements after 30 years. IFCO’s prediction is 0.37 m*

The results for the pore pressures are shown in Figure 4.5 and Figure 4.6 for case 1. Since there are many lines in these graphs, the corresponding cases are shown in the annexes A1_1 till A1_6 and A6_1 till A6_7. The experimental data are drawn as black lines with markers. Solid and open markers refer to a different location. MSettle results are displayed as grey lines. The thickness of the lines refers to the depth; a thick line corresponds to a transducer that lies deeper than the transducer shown by a thin line.

While the settlement is susceptible for both mechanical and hydraulic properties of the subsoil, the pore pressures are mainly susceptible for the hydraulic properties.
Figure 4.1 Section A1 – Calculated and measured settlements for cases with an uncertainty in hydraulic properties

Figure 4.2 Section A1 – Calculated and measured settlements for cases with an uncertainty in mechanical properties
Figure 4.3  Section A6 – Calculated and measured settlements for cases with an uncertainty in hydraulic properties

Figure 4.4  Section A6 – Calculated and measured settlements for cases with an uncertainty in mechanical properties
Figure 4.5  Section A1 – Calculated and measured pore pressures for case 1

Figure 4.6  Section A6 – Calculated and measured pore pressures for case 1
5 Discussion

5.1 IFCO method

Application of the IFCO method involves techniques that make it difficult to assess the settlement since the start of all activity in the field. IFCO’s consultancy is aimed at the most economic use of resources like pumps. It uses differences in settlement and pore pressure in order to check whether the criterion to stop forced consolidation in a section, is met. So the method does not need absolute settlement. This fact is rather unfortunate for our comparison between measurements and calculations: the values of the initial pore pressures and the initial height of the ground level is missing. Therefore we had to device a scheme to compare monitoring data with our calculations.

![Diagram of settlement and measurement data with offset](image)

*Figure 5.1 Calculation and measured data. Offset for settlement that was not recorded*

The calculations start at the moment a trench for the foundation of the runway is dug. This will lead to some heave of the ground level, see Figure 5.1. Monitoring of settlement and pore pressures starts a few weeks later. In the mean while a lot of loading and unloading has taken place, which has its influence on pore pressures and settlement. The exact influence in the field remains unknown. Nevertheless calculations can predict which effects these actions are likely to have on settlement and pore pressures. The assumption in the comparison is that the calculation provides the best guess for the prediction for that part of the settlement that has not been recorded.

IFCO sets the first measured value of the settlement to zero. In order to compensate for the amount of settlement that IFCO has not recorded, we add an offset to the measured settlement. The value of this offset is equal to the difference between the calculated value and the measured value (zero by IFCO’s definition) at the date the first measured settlement is available, see Figure 5.1. The first measured point is translated onto the calculated curve. Each open marker is shifted by an amount of “offset”.

The disadvantage of this method is that differences between calculation and measurement in the first stages of the construction remain hidden. This is inherent to the limited availability of data. What can be achieved is a validation of the model with the available data. By choosing to set the first measured
settlement equal to the corresponding settlement, the shape of the settlement curve is partly validated. As long as there is still a lot of settlement to follow after the date of the first available measurement, such a validation does have its value. However there is a disadvantage. Specially shortly after a change in loading the shape of the settlement curve is rather steep. That means that an uncertainty in the date of the change in load has a large influence in the value of the calculated settlement. This has repercussions for "offset" in Figure 5.1. Since all measured data are shifted by offset, an uncertainty in the time scheme will have large consequences for the degree of agreement between measurement and calculation.

5.2 Settlement
In both sections roughly 0.1 m of settlement is reported. Case 1 represents the best guess for the parameters of soil and load. Other cases reflect the influence of the uncertainty in parameters like volumetric weight and heads. The observed settlement is relatively small: 0.1 m. In [den Adel 2003] values of roughly 2 m are reported.

5.2.1 Section A1
In Figure 4.1 can be clearly seen that the agreement between our best guess (case 1) and the measured settlements is very good, except for the last points where the prediction is a little more than observed. The influence of the head in the Pleistocene sand is large compared to the influence of the head in the compressible layers. For case 4 the agreement between measurement and calculation is very good, when the observed scatter in settlement is taken into account.

In Figure 4.2 the effect of uncertainty in the mechanical loading is visualised. For case 2, the agreement is very good. This case reflects the influence leakage at ground level has on the settlement. Oral communications informs about initial difficulties to reach a low pressure in the drains. This is an indication for leakage. The most probable source for leakage is the foil at ground level. Since the drains are at the bottom of the drainage trenches, a pressure gradient may be created. That means that the air pressure in the sand under the foil may not be the same as in the drain. The effect leakage has on settlement is however smaller than the effect of the head in the Pleistocene sand.

The final settlement predicted by MSettle is 0.205 m for case 1, which means 18 % less than the IFCO prediction. The relative differences vary from 11 to 38 %, depending on the case.

5.2.2 Section A6
In Figure 4.3 the calculated and measured settlements are plotted. Case 1, the best guess, predicts too much settlement, roughly one third more than observed. The influence of the head in the Pleistocene sand is present, and for case 4 the agreement between measurement and calculation becomes better.

Figure 4.4 shows a similar behaviour. The air pressure in the sand has a large influence since for case 2 the agreement is very good. The influence of the date where the second load is applied is not significant.

The final settlement predicted by MSettle is 0.307 m for case 1, which means 17 % less than the IFCO prediction. The relative differences vary from 11 to 34 %, depending on the case.

5.2.3 Both sections
The agreement between measurement and calculation is good for sections A1 and A6, considering the fact that uncertainties in parameters of soil and load have a large influence on settlement.
5.3 Pore pressures

5.3.1 Comparison
The agreement between the measured and calculated pore pressures is weak. Transducers W1 and W4 are at the same depth. W2 with W5 and W3 with W6 form similar pairs. The vertical distance between the pairs of transducers are more or less equal. If the permeability of the layers in which the transducers are located, are roughly the same, the difference between the value of the pore pressure of pairs of transducers should be the same. For section A6 the differences are more or less equal, for section A1 they differ considerably. The differences for section A1 may be attributed to the permeability of the layers in which the transducers are located. All transducers in section A6 are in the layer consisting of clay. For section A1 the top most transducer is located in a sandy layer, whereas the others are in the same type of clay layer as for section A6. The permeability of the sandy layer is less than the permeability of the clay layer. As a result the pressure difference between the deepest transducers and the central transducers should be less than the difference between the topmost transducers and the central transducers. Figure 3.11 shows the opposite. Since initial values on pore pressures are not available, it is not feasible to determine the origin of the observed deviations.

The calculated pore pressures are shown in Figure 4.5 and Figure 4.6 for sections A1 and A6 respectively. Both figures relate to case 1. For section A1 the agreement between calculated pore pressures and measured pore pressures is poor at the beginning as the observed pressure span is twice as large as the calculated pressure span. At the end of pumping phase the agreement is quite good, for sensors at depth –9.50 m and –10.25 m NAP. For section A6 the initial agreement is slightly better. Unfortunately the calculations predict a decrease in pore pressures, whereas the observed pore pressures increase. The other cases (2 until 7) do not achieve a substantial improvement of the agreement. Since Figure 4.5 and Figure 4.6 shown many lines, each case has his own separate annex. Annexes are numbered A1_number and A6_number, where number refers to the number of the case.

Since pore pressures are not recorded during excavation and the several steps that have led to the embankment, the response of the soil on these loads is not known. Such information is vital to assess the permeability of the sub soil. The effect of virtual loads, being the result of the pressure fluctuations in the drainage system, on the calculated pore pressures is rather small. The measured pore pressures do not reflect any of these fluctuations.

5.3.2 MSettle pore pressures predictions
In case of vertical drainage, MSettle solves the average head between two drains using a modified storage equation. A consequence is that, at the position between two drains, the head calculated by MSettle will be underestimated compared to the real head, and the head calculated by MSettle will be overestimated near the drain, as shown in Figure 5.2.
So, it is expected that pore pressures calculated by MSettle are lower than pore pressures measured. This expectation is confirmed by the measurements. Nevertheless, it is not possible to evaluate this underestimation as each pore pressure measurement was performed in the middle between two adjacent drains.
Figure 5.2  Modelling of the head distribution in MSettle in case of vertical drainage
6 Conclusions

- The agreement between measured and calculated settlement is very good for section A1.

- For section A6, there is a difference of about 8 cm on a settlement of 23 cm at the end of the pumping phase, for case 1.

- The differences between the predictions for the final settlement after 30 years are small, using the Isotach model and the IFCO extrapolations. The relative difference is only 18 and 17% respectively for section A1 and A6. Since the final settlement will be known in about 30 years, a conclusion like “model A is better than model B” can not yet be made.

- The agreement between measurement and calculation is better for settlement than for pore pressures. A similar result has been found in [den Adel 2003].

- The value of the absolute settlement was not measured. This was not an omission of the contractor, since consultancy related to the IFCO method uses changes in settlement and pore pressures. However the absence implies some limitations on the comparison between measurement and calculations, as described in this report.

- The time schedule for section A6 is not known in detail. Although using a similar time schedule as in section A1 is the best possible estimation, this assumption will introduce some uncertainty in the results.

- Various uncertainties in load and strength contribute to variations in the differences between measurement and calculation. Their influence has been reported in the different cases. From these cases is concluded that the head in the Pleistocene sand and the air pressure in the sand under the foil are major contributors.

- The agreement between measured pore pressures and calculated values is poor. Since initial values of pore pressures are not available, the origin of the observed deviations can not be explained in a reliable way. The fact that the average pore pressure is calculated, will contribute to the differences between calculation and measurement.

- In which extent the concept of the Isotach model contributes to the observed differences between measurement and calculation, is hard to determine. The global idea is that uncertainties in load and strength do have a major influence on the predictions of the model. If these uncertainties can be narrowed to a minor margin, the deviations between predictions and observations will definitely decrease.

- This report has shown that for the sections A1 and A6 of the 5th runway, where a method of intensified consolidation (like IFCO) is used, the Isotach model is a good predictor for the settlement. It should be noted that two sources of uncertainty have complicated the comparison between measured and calculated results: the fluctuations in the observed settlement and the absence of measured data in the initial situation.
7 Recommendations

- If the Isotach model is to be used in situations where an intensified or forced system of drainage is a vital part of the construction process, a monitoring system for pore pressures and settlement is of importance.

- Measurement of and reporting the initial situation will simplify the understanding and interpretation of monitoring data.

- Detailed information on the boundary conditions, like the head in the Pleistocene sand, is necessary to extrapolate or calculate the (remaining) long term settlement.

- When applying the IFCO method for forced drainage information on the magnitude of the pressure drop over the sealing foil will enhance the accuracy of the (remaining) long term settlement.

- Detailed information on the volumetric weight of the soil in the embankment will increase the reliability of the predictions.

- The influence of fluctuations of the hydraulic head in the pleistocene sand on the final settlement should be calculated or estimated. This will determine the sense or nonsense of very accurate predictions of the settlement after 30 years.
8 Linear and natural strain

The $K_0$-CRS test is performed at a constant rate of strain, i.e.:

$$\varepsilon = E \times t$$ \hspace{1cm} (8.1)

Where $\varepsilon$ is the linear strain, $t$ is time and $E$ is the rate by which the linear strain increases. The relation between linear an natural strain is:

$$\varepsilon^H = -\ln(1 - \varepsilon)$$ \hspace{1cm} (8.2)

Where $\varepsilon^H$ is the natural strain. Differentiate equation (8.2) with respect tot time:

$$\frac{\partial \varepsilon^H}{\partial t} = -\frac{1}{1 - \varepsilon} \times \frac{\partial \varepsilon}{\partial t} = \frac{1}{1 - Et} \times E = \frac{(1 + Et)E}{1 - Et}$$ \hspace{1cm} (8.3)

So when the linear rate of strain is constant ($E$), the rate of natural strain increases with time (or strain). The approximation ($\approx$) is valid for small values of the natural strain: $\varepsilon < 0.2$.

---

Figure 8.1 The stress natural strain diagram

In Figure 8.1 the relation between natural strain and stress is drawn. When the rate of strain increases, the line, describing the relation between stress and natural strain, shifts towards higher stresses. So when the natural strain is increasing, the relation between stress and strain is not parallel to the lines in Figure 8.1. The line is comparable to the dashed line. A buckle in the line, like seen in Figure 3.14 is not predicted.

The effect is small. The distance between the two Isotach lines is $b \times c$. The value of $b$ is of the order of 0.1, $c$ is roughly 0.01. Their product is 0.001. That means that the value of $\sigma_2$ is 0.1% larger than $\sigma_1$. 

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Since the change in rate of natural strain increases roughly 25%, the value of $\sigma_2$ is just 0.01% larger than $\sigma_1$. Such a small difference is negligible.
9 References

den Adel, H.
Uitwerking K₀-CRS proef, bepaling abc parameters
Delft Cluster report, 01.04.02, March 2002 (In Dutch)

den Adel, H., and Trompille, V.
Validation Isotach model by means of “Barendrechtse weg”

den Haan, E.J.
Vertical compression of soils

den Haan, E.J., and Sellmeijer, J.B.
Calculation of soft ground settlement with an Isotache method.
A.S.C.E. Geotechnical Special Publication nr. 112, 94-104.

den Haan, E.J.
Voorspelling restzettingen met het abc isotachenmodel
Delft Cluster report, 01.04.02, June 2002, (In Dutch)

Sellmeijer, J.B.
Isotachenmodel bij lage spanningen, onderdrukken numerieke oscillaties
Delft Cluster report, 01.04.02, March 2002 (In Dutch)
General Appendix: Delft Cluster Research Programme Information

This publication is a result of the Delft Cluster research-program 1999-2002 (ICES-KIS-II), which consists of 7 research themes:

- Soil and structures
- Risks due to flooding
- Coast and river
- Urban infrastructure
- Subsurface management
- Integrated water resources management
- Knowledge management

This publication is part of:

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Delft Cluster is an open knowledge network of five Delft-based institutes for long-term fundamental strategic research focussed on the sustainable development of densely populated delta areas.

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**Theme Management team: Ground and Construction**

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<tr>
<td>Dr. P. van den Berg</td>
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<td>Dr. J. Rots</td>
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**Project group**

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<td>1 Dr. H. den Adel</td>
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<td>3 Dr. A. Scarpas</td>
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<td>4 Ir. P. Waarts</td>
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**Other Involved personnel**

The realisation of this report involved:

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ANNEXES
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11
1 - Boundary number
   1 - number of curves on boundary, next line(s) are curvenumbers
   3
2 - Boundary number
   1 - number of curves on boundary, next line(s) are curvenumbers
   4
3 - Boundary number
   1 - number of curves on boundary, next line(s) are curvenumbers
   5
4 - Boundary number
   1 - number of curves on boundary, next line(s) are curvenumbers
   6
5 - Boundary number
   1 - number of curves on boundary, next line(s) are curvenumbers
   7
6 - Boundary number
   1 - number of curves on boundary, next line(s) are curvenumbers
   8
7 - Boundary number
   1 - number of curves on boundary, next line(s) are curvenumbers
   9
8 - Boundary number
   1 - number of curves on boundary, next line(s) are curvenumbers
  10

[PIEZOElectric LINES]
2 - Number of piezometric level lines -
   1 - PILine number
      1 - number of curves on PILine, next line(s) are curvenumbers
      1
   2 - PILine number
      1 - number of curves on PILine, next line(s) are curvenumbers
      2

[PHREATIC LINE]
1 - Number of the piezometric level line acting as phreatic line -

[WORLD CO-ORDINATES]
0.000 - X world 1 -
0.000 - Y world 1 -
0.000 - X world 2 -
0.000 - Y world 2 -

[LAYERS]
8 - Number of layers -
   1 - Layer number, next line is material of layer
      Peat (60)
      1 - Piezometric level line at top of layer
      2 - Piezometric level line at bottom of layer
      1 - Boundary number at top of layer
      0 - Boundary number at bottom of layer
   2 - Layer number, next line is material of layer
      Clay, silty (59b)
      1 - Piezometric level line at top of layer
      1 - Piezometric level line at bottom of layer
      2 - Boundary number at top of layer
      1 - Boundary number at bottom of layer
   3 - Layer number, next line is material of layer
      Clay, silty (58)
      1 - Piezometric level line at top of layer
      1 - Piezometric level line at bottom of layer
      3 - Boundary number at top of layer
      2 - Boundary number at bottom of layer
   4 - Layer number, next line is material of layer
      Clay, very silty (57)
      1 - Piezometric level line at top of layer
      1 - Piezometric level line at bottom of layer
4 - Boundary number at top of layer
3 - Boundary number at bottom of layer
5 - Layer number, next line is material of layer
    Sand (B09 - sample 3)
    1 - Piezometric level line at top of layer
    1 - Piezometric level line at bottom of layer
    5 - Boundary number at top of layer
    4 - Boundary number at bottom of layer
6 - Layer number, next line is material of layer
    Sand clayey (56a)
    1 - Piezometric level line at top of layer
    1 - Piezometric level line at bottom of layer
    6 - Boundary number at top of layer
    5 - Boundary number at bottom of layer
7 - Layer number, next line is material of layer
    Sand (B09 - sample 3)
    1 - Piezometric level line at top of layer
    1 - Piezometric level line at bottom of layer
    7 - Boundary number at top of layer
    6 - Boundary number at bottom of layer
8 - Layer number, next line is material of layer
    Clay, silty (53)
    1 - Piezometric level line at top of layer
    1 - Piezometric level line at bottom of layer
    8 - Boundary number at top of layer
    7 - Boundary number at bottom of layer

[END OF GEOMETRY DATA]
[RUN IDENTIFICATION]
Schiphol project - Section A1
Case 1: Basic situation
[MODEL]
1 : Dimension = 2D
0 : Calculation type = Darcy
2 : Model - Isotache
1 : Strain type = Natural
1 : Vertical drains = TRUE
1 : Fit for settlement plate = TRUE
[VERTICALS]
    100 = total Mesh
    2 = number of items
    0.000 0.000 = X, Z
    32.500 0.000 = X, Z
[WATER]
    9.81 = Unit Weight of Water
    20000000 = Bulk Modulus of Water
[NON-UNIFORM LOADS]
    4 = number of items
1: digging
    0 -14.50 -18.70 = Time, Gamma dry, Gamma wet
11 = Number of co-ordinates
-37.500 -4.850 = X, Y
-37.500 -5.025 = X, Y
-30.000 -5.125 = X, Y
-30.000 -5.625 = X, Y
-15.000 -5.625 = X, Y
0.000 -5.400 = X, Y
15.000 -5.625 = X, Y
30.000 -5.625 = X, Y
30.000 -5.125 = X, Y
37.500 -5.025 = X, Y
37.500 -4.850 = X, Y
2a: pre-load (foundation)
    12 20.00 20.00 = Time, Gamma dry, Gamma wet
2 = Number of co-ordinates
-37.500 -4.850 = X, Y
37.500 -4.850 = X, Y
2b: pre-load (until foil)
12 20.00 20.00 = Time, Gamma dry, Gamma wet
7 = Number of co-ordinates
-38.188 -4.850 = X, Y
-37.963 -4.675 = X, Y
-15.000 -4.675 = X, Y
0.000 -4.450 = X, Y
15.000 -4.675 = X, Y
37.963 -4.675 = X, Y
38.188 -4.850 = X, Y
3: pre-load (road)
39 20.00 20.00 = Time, Gamma dry, Gamma wet
5 = Number of co-ordinates
-37.963 -4.675 = X, Y
-37.500 -4.212 = X, Y
0.000 -3.650 = X, Y
37.500 -4.212 = X, Y
37.963 -4.675 = X, Y
[WATER LOADS]
0 = number of items
[OTHER LOADS]
2 = number of items
t=109 Nil load
3: Uniform
109 0.01 0.001 -12.700 = Time, Gamma, H, Yapplication
[t=165 Nil load
3: Uniform
165 0.01 0.001 -12.700 = Time, Gamma, H, Yapplication
[CALCULATION OPTIONS]
5 : Precon. pressure within a layer = Variable, correction at every step
0 : Imaginary surface = FALSE
1 : Submerging = TRUE
0 : Maintain profile = FALSE
Superelevation
0 = Time superelevation
10.00 = Gamma dry superelevation
10.00 = Gamma wet superelevation
1 : Dispersion conditions layer boundaries top = Drained
1 : Dispersion conditions layer boundaries bottom = Drained
0 : Stress distribution soil = Buisman
1 : Stress distribution loads = Simulate
0.10 = Iteration stop criteria submerging [m]
0.10 = Iteration stop criteria desired profile [m]
1.00 = Load column width imaginary surface [m]
1.00 = Load column width non-uniform loads [m]
1.00 = Load column width trapeziform loads [m]
10000 = End of consolidation [days]
8 = Number of subtime steps
1.000E+00 = Reference time
[RESIDUAL TIMES]
0 : Number of items
[PORE PRESSURE METERS]
0 = number of items
[NON-UNIFORM LOADS PORE PRESSURES]
4 = number of items
1: digging
0.000 = Top of heightening
2a: pre-load (foundation)
0.000 = Top of heightening
2b: pre-load (until foil)
0.000 = Top of heightening
3: pre-load (road)
0.000 = Top of heightening
[OTHER LOADS PORE PRESSURES]
2 = number of items
t=109 Nil load
0.000 = Top of heightening
t=165 Nil load
0.000 = Top of heightening
[CALCULATION OPTIONS FOR PRESSURES]
1 : Shear stress = TRUE
1 : calculation method of lateral stress ratio (k0) = Nu

[VERTICAL DRAIN]
1 : Flow type = Plane
-10.200 = Bottom position
-10.075 = Position of the drain pipe
3.500 = Centre to centre distance
0.250 = Diameter
12 = number of items

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<th>Water level</th>
<th>Tube</th>
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<td>Time</td>
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<td>Water level</td>
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[1D GEOMETRY]
0.000 = Phreatic level
0.000 = Bottom level
0 = Number of layers

[END OF INPUT FILE]
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<td>With chunks of clay</td>
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<td>CLAY, moderately silty, slightly peaty</td>
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<td>With some remains of plants</td>
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<tr>
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<td>-0.60</td>
<td>-1.05</td>
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<td>With layers of sand</td>
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<td>-1.85</td>
<td>SAND, moderately silty</td>
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<td>With some shells fragments</td>
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<td>M = 150 μm</td>
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<td>-3.38</td>
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<td>With chunks of clay</td>
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<td>With broken shells</td>
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<td>-3.38</td>
<td>-4.62</td>
<td>M = 150 μm</td>
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<tr>
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<td>-4.62</td>
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<td>With broken shells</td>
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<td>-7.05</td>
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<td>With remains of plants</td>
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<td>With some chunks of loam</td>
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<td>-8.65</td>
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<td></td>
<td>M = 150 μm</td>
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<td>-9.15</td>
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<td>M = 210 μm</td>
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<td>-9.50</td>
<td>SAND, slightly silty</td>
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<td>rich on lime:</td>
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<td>from GL -0.10 till GL -7.43 m</td>
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<td>from GL -9.15 till GL -9.50 m</td>
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<td>without lime:</td>
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<td>from GL -7.43 till GL -9.15 m</td>
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END OF BOREHOLE 01
Geometry of the sub soil in section A1: 3600-3800
Schematization as used in the calculations
ISOTACH model 5th runway SCHIPHOL
Geometry of the sub soil in section A6: 2400-2600
Schematization as used in the calculations
ISOTACH model 5th runway SCHIPHOL
Figure: Pore pressures

Section A1 - Case 1

- Measurement: depth -11.00 m NAP, km 3650
- Measurement: depth -11.00 m NAP, km 3750
- Measurement: depth -10.25 m NAP, km 3650
- Measurement: depth -10.25 m NAP, km 3750
- Measurement: depth -9.50 m NAP, km 3650
- Measurement: depth -9.50 m NAP, km 3750
- MSettle 6.7: depth -11 m NAP
- MSettle 6.7: depth -10.2 m NAP
- MSettle 6.7: depth -9.485 m NAP
- Pressure in the pipe

Date:
- 23-Mar-01
- 22-Apr-01
- 22-May-01
- 21-Jun-01
- 21-Jul-01
- 20-Aug-01
- 19-Sep-01
Schiphol project
Section A1, case 3
Pore pressures
Section A1 - Case 4

Water pressure [kPa]

Pressure in the pipe [kPa]

Date


Schiphol project
Section A1, case 4
Pore pressures
Schiphol project
Section A1, case 5
Pore pressures

![Graph showing pore pressures over time, with various measurement points and time stamps from March 2001 to September 2001.](image-url)
Pore pressures
Section A6 - Case 2

Water pressure [kPa]

Pressure in the pipe [kPa]

Date

3-May-01  2-Jun-01  2-Jul-01  1-Aug-01  31-Aug-01  30-Sep-01  30-Oct-01

Measurement: depth -11.00 m NAP, km 2450
Measurement: depth -11.00 m NAP, km 2550
Measurement: depth -10.25 m NAP, km 2450
Measurement: depth -10.25 m NAP, km 2550
Measurement: depth -9.50 m NAP, km 2450
Measurement: depth -9.50 m NAP, km 2550
MSettle 6.7: depth -10.933 m NAP
MSettle 6.7: depth -10.244 m NAP
MSettle 6.7: depth -9.359 m NAP
Pressure in the pipe

Pore pressures

Schiphol project
Section A6, case 2

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2003-03-17

data
Trp

2003-03-17

CO710402

ctt.
Adel

AnnexA6_2

form.
A4
Pore pressures
Section A6 - Case 4

- Measurement: depth -11.00 m NAP, km 2450
- Measurement: depth -11.00 m NAP, km 2550
- Measurement: depth -10.25 m NAP, km 2450
- Measurement: depth -10.25 m NAP, km 2550
- Measurement: depth -9.30 m NAP, km 2450
- Measurement: depth -9.30 m NAP, km 2550
- MSellite 6.7: depth -10.933 m NAP
- MSellite 6.7: depth -10.244 m NAP
- MSellite 6.7: depth -9.529 m NAP
- Pressure in the pipe

Water pressure [kPa] vs. Date

- 3-May-01
- 2-Jun-01
- 2-Jul-01
- 1-Aug-01
- 31-Aug-01
- 30-Sep-01
- 30-Oct-01

Pressure in the pipe [kPa]
Section A6 - Case 5

Water pressure [kPa]

- Measurement: depth -11.00 m NAP, km 2450
- Measurement: depth -11.00 m NAP, km 2550
- Measurement: depth -10.25 m NAP, km 2450
- Measurement: depth -10.25 m NAP, km 2550
- Measurement: depth - 9.50 m NAP, km 2450
- Measurement: depth - 9.50 m NAP, km 2550
- MSelte 6.7: depth -10.933 m NAP
- MSelte 6.7: depth -10.244 m NAP
- MSelte 6.7: depth -9.529 m NAP
- Pressure in the pipe

Pressure in the pipe [kPa]

Date

3-May-01 2-Jun-01 2-Jul-01 1-Aug-01 31-Aug-01 30-Sep-01 30-Oct-01

Schiphol project
Section A6, case 5
Pore pressures
Section A6 - Case 7

Water pressure [kPa]

Pressure in the pipe [kPa]

Date

24-May-01 23-Jun-01 23-Jul-01 22-Aug-01

Pore pressures
Geotechnical profile of section A1: 3600-3800
Based on the Tjaden report
ISOTACH model 5th runway SCHIPHOL